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Investigation of a Steel Viaduct

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INVESTIGATION OF A STEEL VIADUCT

BY

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THESIS

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THIS IS TO CERTIFY THAT THE THESIS PREPARED UNDER MY SUPERVISION BY

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ENTITLED INVESTIGATION OF A STEEL VIADUCT

IS APPROVED BY ME AS FULFILLING THIS PART OF THE REQUIREMENTS FOR THE

DEGREE OF Bachelor of Science in Civil Engineering

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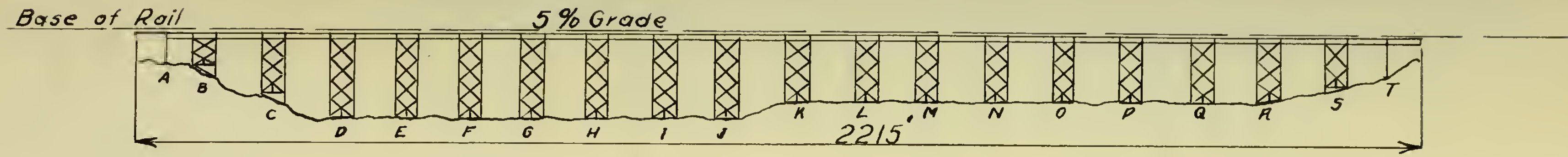


Fig. 1

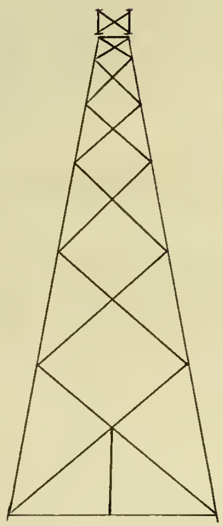


Fig. 2

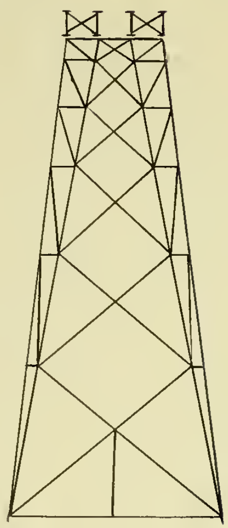


Fig. 3

RICHLAND CREEK VIADUCT

I.

GENERAL DESCRIPTION OF RICHLAND CREEK VIADUCT.

The Richland Creek viaduct crosses Richland Creek on the line of the Indianapolis Southern Railroad in Greene County, Indiana, seven miles from Bloomfield, on a tangent at a 5 per cent grade, with a steel superstructure 2215 feet in length and 132 feet in maximum height. It is a single track deck girder bridge, having eighteen alternate 40- foot tower spans and twenty-one 75-foot viaduct spans, besides two 60- and two 50-foot spans at the upper and lower ends supported by single rocker bents next to the abutments. The girders have a uniform depth of seven feet, are spaced eight feet apart transversley, and are connected by diagonal lateral angles in the plane of the upper and lower flanges and by vertical sway brace frames.

The towers vary in height from 65 to 125 feet and have rectangular bases except the last two at each end, which have trapezoidal bases owing to the rise of the ground. The bents are vertical and spaced 40-feet apart. Each one consists of two columns, battered 2 1/4 in. in 20 in., of a rectangular cross-section composed of two pairs of 4" x 4" angles x 9/16" above the splice and x 5/8" below, two 21-inch web-plates and #2 x 2" x 3/8" angle-lacing. The transverse as well as the longitudinal bracing being stiff members made up of 10-inch 20-pound channels and 2 1/2" x 3/8" double lacing: no horizon-

tal struts are used except at the top and bottom: and these struts are supported at their centers by hangers which reach to the intersection of the diagonals in the panel above. They are made up of two pairs of angles 3 1/2 " x 3" x 3/8" with single lacing at 60 degrees. All the columns ninety-one feet or more in height are spliced in two sections, all others being in one section. The feet of opposite diagonal columns in each tower have expansion bearings with a 3/8-inch phosphor bronze plate sliding on a 7/8-inch steel masonry plate; all other columns have fixed bearings and rest on 1 1/4" steel masonry plates. Horizontal cap plates at the top of each column receive the ends of the longitudinal girders, which have expansion bearings at one end of the viaduct spans.

The structure is designed for Cooper's E50 live load and provision was made for impact stresses, the impact formula used is,

$$I = LL \times \frac{LL}{DL + LL}$$

where I, impact stress,

DL, dead load stress,

LL, live load stress.

For tension the unit stresses used, were 15000 and 19000 pounds per square inch; for compression,

$$\frac{15000}{1 + \frac{l^2}{13500r^2}} \quad \text{and} \quad \frac{19000}{1 + \frac{l^2}{13500\ r^2}}$$ and for shear a

maximum of 9000 pounds per square inch and 22000 per square inch respectively were used for shop rivets, and 8800 and 17600

pounds per square inch for field rivets. All material is open hearth steel. The viaduct is now used for single track, but was designed so that it could be converted into a double track viaduct by the addition of auxilliary columns and bracing as ~~a~~ shown in Plate I, Fig. 3.

The superstructure contains 2098 tons of steel fabricated at the Detroit plant of the American Bridge Company and erected by the Strobel Stub Construction Company of Chicago. The design and construction was in charge of R. E. Grant, Engineer of ~~of~~ Bridges of the Illinois Central Railroad.

II.

DETERMINATION OF WEIGHT OF STEEL IN RICHLAND CREEK VIADUCT.

The weight of steel was determined in detail from the drawings, and the following tables give the results. The letters, A, B, C, etc., of first table, refer to the different towers of the viaduct, see Plate I. The second table gives the weights of the girders.

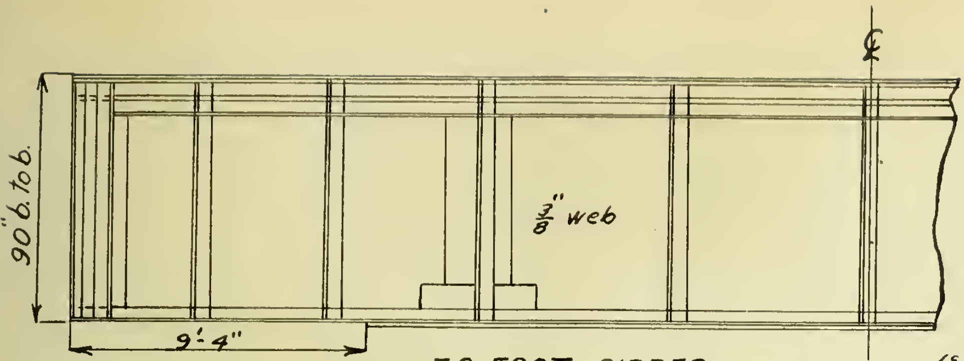
TABLE I.

Weight of Towers.

No.	Description.	Weights. lbs.	Total Weight. lbs.
7	D,E,F,G,H,I,J,	200000	1400000
4	K,L,M,N,	125000	500000
3	O,P,Q,	100000	300000
1	B	64000	64000
1	C	128000	128000
1	R	98000	98000
1	S	30000	30000
1	A	6000	6000
1	T	15000	15000

Total weight in towers 2480000 lbs.

PLATE II



50 FOOT GIRDER

Fig. 1

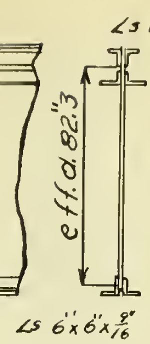


Fig. 2

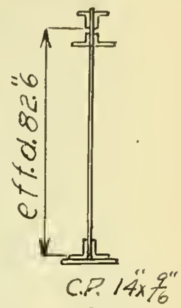
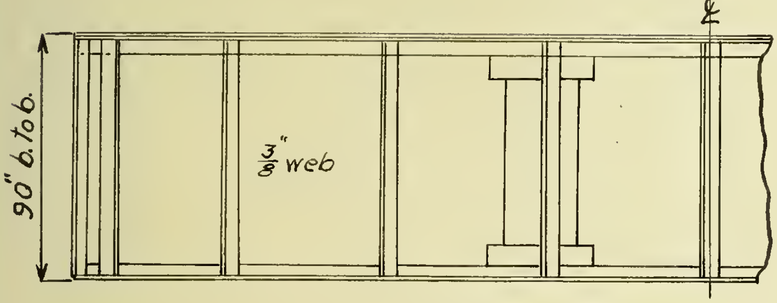


Fig. 3



40 FOOT GIRDER

Fig. 4

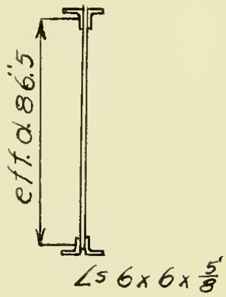
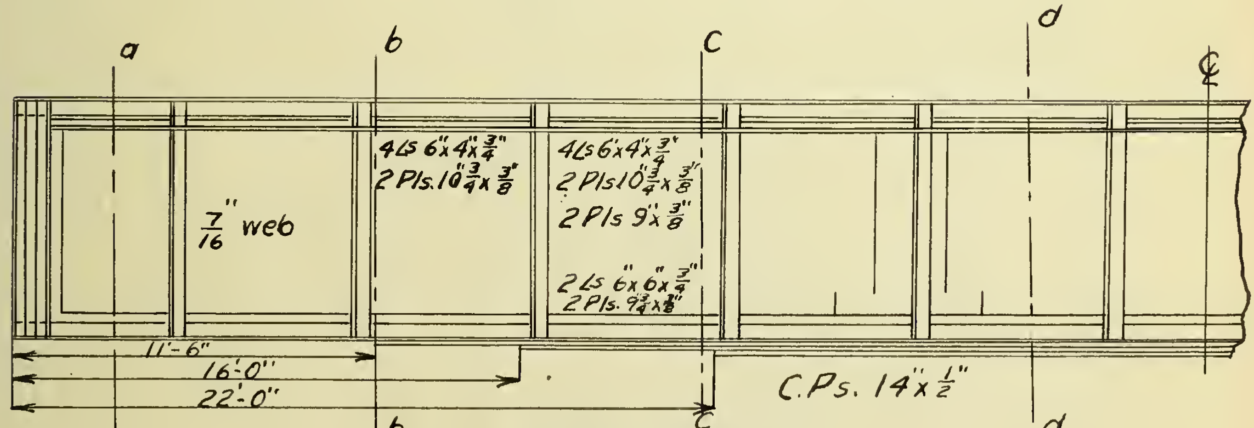
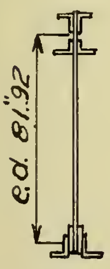


Fig. 5



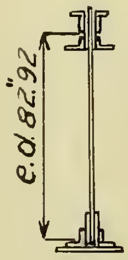
75 FOOT GIRDER

Fig. 6



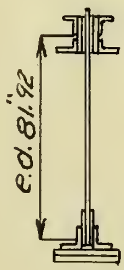
Sect. a-a

Fig. 7



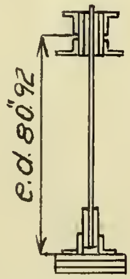
sect. b-b

Fig. 8



sect. c-c.

Fig. 9



sect. d-d

Fig. 10

TABLE II.

Weight of Girders.

No.	Description.	Weights. lbs.	Total Weight. lbs.
17	75 feet.	70000	1119000
18	40 feet.	20000	360000
2	50 feet.	30000	60000
2	60 feet.	39000	80000

Total weight in girders, 1690000 lbs.

Grand total weight, 4,170,000 lbs. = 2085 tons.

III.

1. STRESSES IN GIRDERS.

The stresses are determined by the ordinary methods of mechanics. Impact stresses are determined from the formula

$$I = LL. \times \frac{LL.}{DL. + LL.}$$

in which, I - impact stress.

LL = live load stress.

DL - dead load stress.

The unit allowable stresses for tension are, 15000 and 19000 pounds per square inch for live and dead loads respectively. One eighth of the web was considered as efficient flange area.

The results with the efficiencies are in tables III to VI inclusive, and see plates III, VI and V, for the shear and moment curves of the above girders.

WEB STRESSES, WEB AREAS, AND EFFICIENCIES. The following table contains all quantities necessary for the determination of the maximum web stresses and the investigation of the web. The ordinary methods in mechanics were used for determining the dead and live load shears. The unit allowable shearing stress is 9000 pounds per square inch. See Plate II for sections of girders.

TABLE III.

Girder	Point	D.L.V.	L.L.V.	Max.V.	Allow.V.	Req.A.	Act.A.	Eff.
40'	0	8600	94000	102600	9000	11.4 "	32.9 "	290%
50'	0	15000	109000	124000	9000	13.8 "	31.3 "	226
75'	0	35000	146000	181000	9000	20.0 "	35.3 "	177

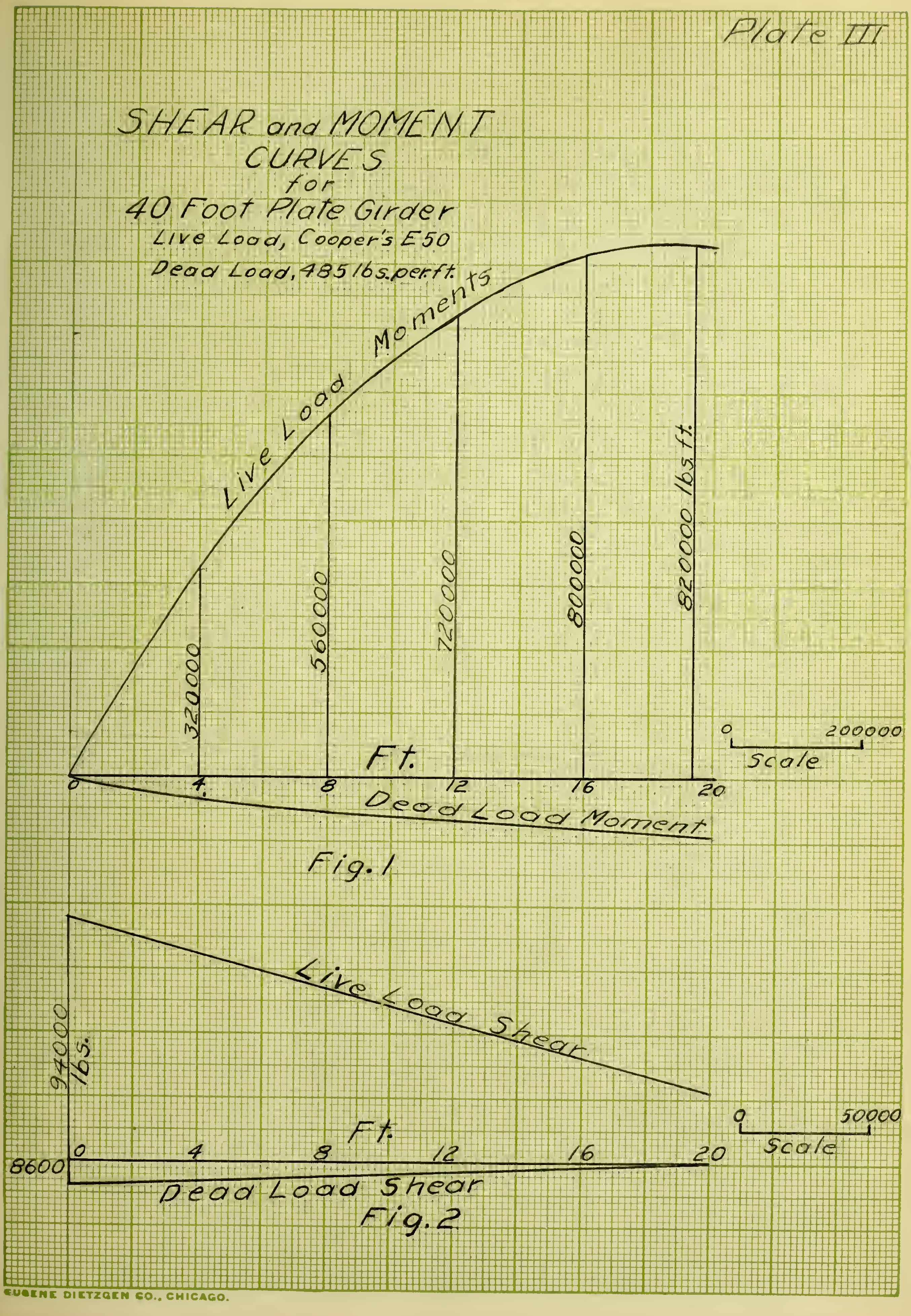


Plate III
SHEAR and MOMENT
CURVES
for
40 Foot Plate Girder
Live Load, Cooper's E50
Dead Load, 485 lbs. per ft.
Live Load Moments
320000
560000
720000
800000
820000 lbs. ft.
0
200000
Scale
Ft.
0
4
8
12
16
20
Dead Load Moment
Fig. 1
Live Load Shear
94000 lbs.
0
50000
Scale
Ft.
0
4
8
12
16
20
8600
Dead Load Shear
Fig. 2
EUGENE DIETZGEN CO., CHICAGO.

Plate IV

SHEAR and MOMENT Curves for 50 Foot Plate Girder

Live Load, Cooper's E50
Dead Load, 600 lbs. per ft.

Fig. 1

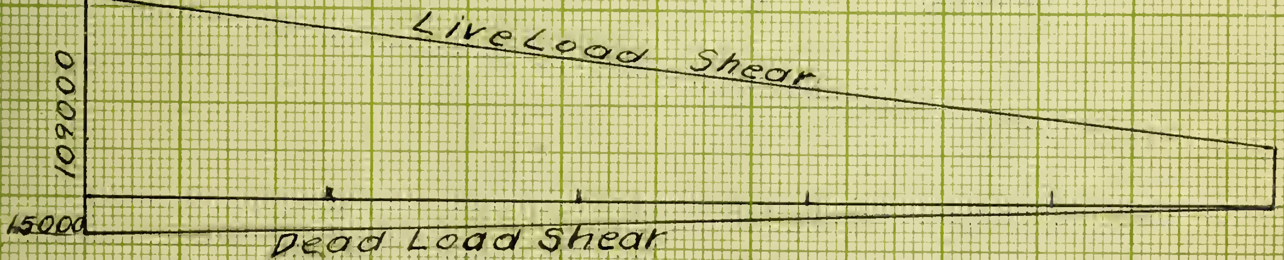
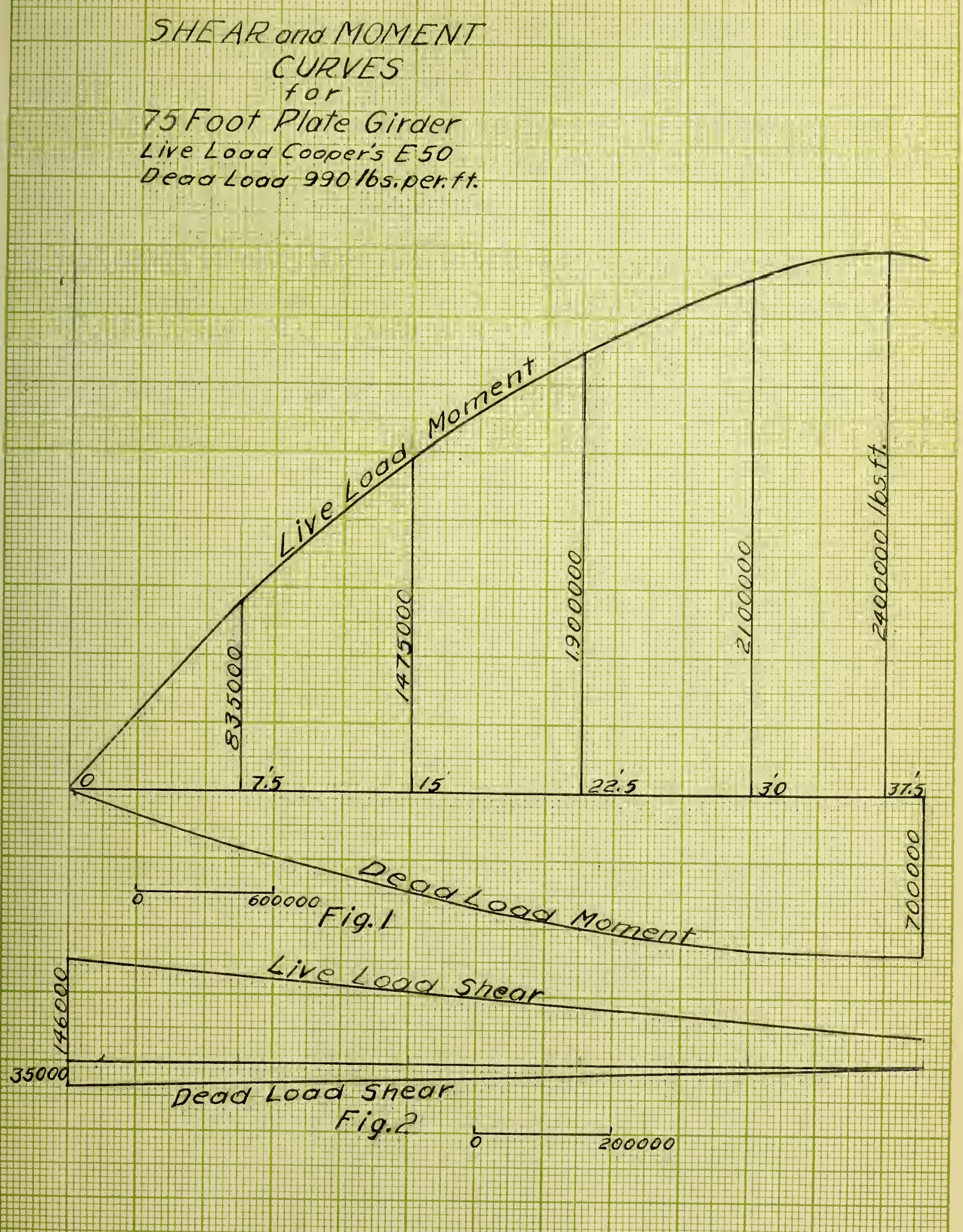


Fig. 2

Plate V


SHEAR and MOMENT
CURVES
for
75 Foot Plate Girder
Live Load Cooper's E 50
Dead Load 990 lbs. per. ft.
Live Load Moment
835000
1475000
1900000
2100000
2400000 lbs. ft.
0
7.5
15
22.5
30
37.5
Dead Load Moment
700000
0
600000
Fig. 1
Live Load Shear
146000
35000
Dead Load Shear
Fig. 2
0
200000

FLANGE STRESSES, AREAS, AND EFFICIENCIES. The following tables contain all quantities necessary for the determination of the flange stresses at the different sections, and all quantities necessary for the investigation of the flanges and their efficiencies. The stresses and efficiencies were calculated at each point where the flange section changed. See Plate II for the sections. The live and dead load moments are given in foot pounds. The allowable unit tensile stresses are 15000 and 19000 pounds per square inch respectively for live and dead loads.

TABLE IV.

40-foot Girder.

Point.	L.L. Mom.	D.L. Mom.	D.L.S.	L.L.S.	Imp't.
Center.	820000	97000	13400	113500	102150

Eff. Depth.	Req. A.	Act. A.	Effic.
86.54"	15.1 "	17.8 "	118%

TABLE V.

50-foot Girder.

Point.	L.L.M.	D.L.M.	D.L.S.	L.L.S.	Imp't.	Eff.D.
Center	1190000	188000	27400	1730000	150000	82.3"
O 9'-4"	680000	120000	17400	99000	84000	82.6"

Req. A.	Act. A.	Effic.
22.94 "	24.5 "	106%
13.1 "	16.76 "	128%

PLATE VI

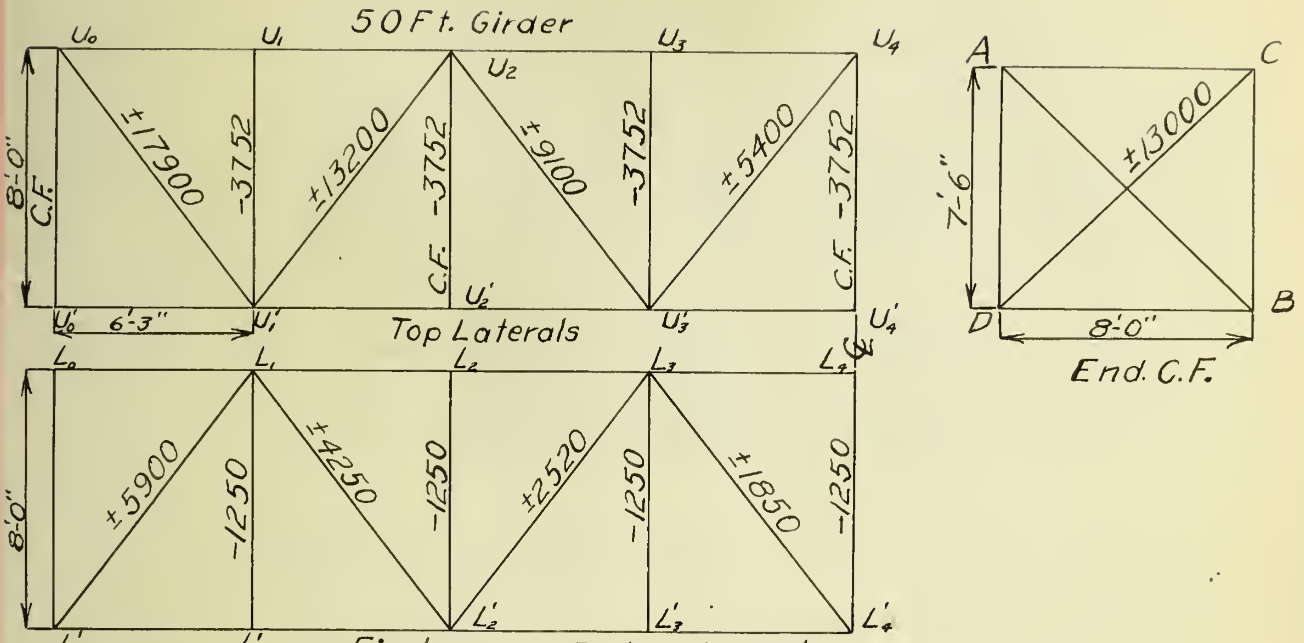


Fig. 1

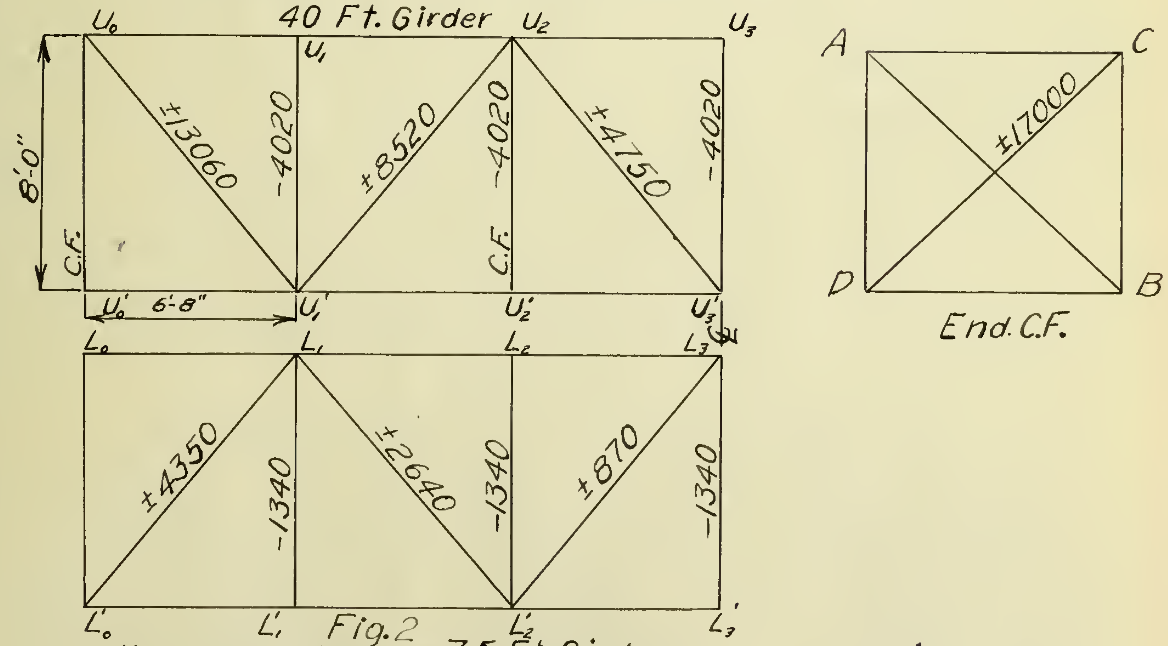


Fig. 2

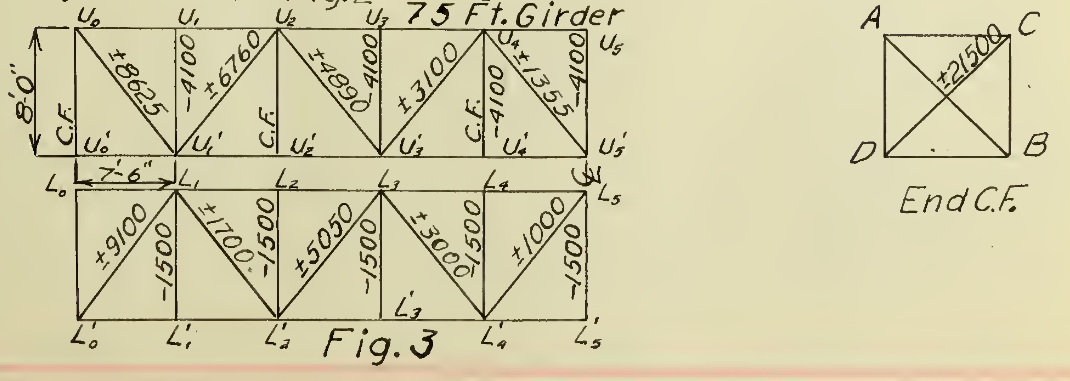


Fig. 3

TABLE VI.

75-foot Girder.

Point	L.L.M.	D.L.M.	D.L.S.	L.L.S.	Imp't	Eff. D.
Center	2400000	700000	103000	350000	270000	81.9"
0 11'-6"	1200000	400000	58000	174000	130000	82.9"
0 16'-0"	1550000	500000	73000	228000	176000	81.9"
0 22'-0"	1750000	650000	96000	260000	190000	80.9"

Req. A.	Act. A.	Effic.
46.7 "	50.3 "	108%
23.7 "	28.4 "	120%
30.8 "	35.7 "	116%
35.0 "	43.0 "	122%

STRESSES IN GIRDER LATERALS. The stresses were determined by the ordinary method of mechanics; the stresses in the upper lateral system were computed for a dead load of 150 pounds per linear foot, and a live load of 450 pounds per linear foot; and in the lower lateral system, stresses were computed for a dead load of 200 pounds per linear foot. Table VII gives the stresses in the laterals of the 40, 50 and 75-foot girders. For strain stresses, see Plate VI.

TABLE VII.

Maximum Stresses in Girder Laterals.

Member	50-foot Girder Stress. lbs.	40-foot Girder Stress. lbs.	75-foot Girder Stress. lbs.
		Top Laterals.	
U_0U_1'	±17900	±13060	±8625
U_1U_1'	-3752 or 0	-4020 or 0	-4100 or 0
U_1U_2'	±13200	±8520	±6760
U_2U_2'	-3752 or 0	-4020 or 0	-4100 or 0
U_2U_3'	±9100	±4570	±4890
U_3U_3'	-3752 or 0	-4020 or 0	-4100 or 0
U_3U_4'	±5400		±3100
U_4U_4'	-3752 or 0		-4100 or 0
U_4U_5'			±1355
U_5U_5'			-4100 or 0

TABLE VII.

(Continued)

Member	50-foot Girder Stress. lbs.	40-foot Girder Stress. lbs.	75-foot Girder Stress. lbs.
	Bottom Laterals.		
$L_0'L_1$	±5900	±4350	±9100
$L_1\ L_1'$	-1250 or 0	-1340 or 0	-1500 or 0
$L_1\ L_2'$	±4250	±2640	±7100
$L_2'L_2$	-1250 or 0	-1340 or 0	-1500 or 0
$L_2'L_3$	±2520	±870	±5050
$L_3\ L_3'$	-1250 or 0	-1340 or 0	-1500 or 0
$L_3\ L_4'$	±1850	.	±3000
$L_4\ L_4'$	-1250 or 0		-1500 or 0
$L_4'L_5$			±1000
$L_5'L_5$			-1500 or 0

2. STRESSES IN BENTS.

DEAD LOAD STRESSES IN TRANSVERSE BENTS. There are no horizontal struts in the transverse bracing and therefore the bracing must consist of stiff members cabable of resisting tension or compression. The triangle is the truss element, and when there are more members than are necessary to form a truss composed of triangles, some member or members must be redundant. This is evidently the case with the truss under consideration. (See figure 2, Plate I.) The stresses in a simple truss are determined by the method of statics, there being the three static equations of equilibrium, but where there is a redundant member or members, each one introduces an unknown quantity in excess of the number determinable by statics.

The determination of these stresses is made possible by means of the theory of least work, which furnishes another equation for equilibrium. The fundamental principle of this

theory is that the true stresses in a redundant system are such that the total internal work of all the stresses is a minimum. Another fundamental proposition is that the external work of deformation by loads is equal to the internal work of resistance. By this proposition the formula $\Delta = \frac{pul}{AE}$ is derived.

where,

Δ , deflection in inches,

p, stress in member,

l, length in inches,

A, area of cross-section in square inches,

E, modulus of elasticity,

u, stress in member for 1 lb. at joint.

The following derivation of this formula is given by J. B. Johnson in the "Theory and Practice of Modern Framed Structures."

The external work of deformation is the product of the load into the movement of the loaded point in the direction of the applied force, divided by two. The internal work of resistance is the sum of the products of the stress produced in each member by its deformation, divided by two.

Thus if W is any external force, acting in any direction, D, the movement of this loaded point in the direction of the force W, P, the total stress produced in any member by the force W, z, the deformation of any member accompanying the stress P, then we have, as the equation between the external and internal work.

$$\frac{WD}{2} = \Sigma \frac{Pz}{2}$$

where the second member represents the algebraic sum of the quantities $\frac{P\,z}{2}$ computed for all the members of the truss for the applied force W.

From the above it would appear that the total internal work is made up of as many parts as there are members in the truss, each are contributing its portion $\frac{P\,z}{2}$ for that member. Similarly, the total deflection D is made up of as many parts as there are members of the truss, these portions d_1, d_2, d_3, etc., being produced by the deformation of the several members.

Whence we may write, from (1);

$$\frac{W\,D_1}{2} = \frac{P_1\,z_1}{2}, \qquad \frac{W\,d_2}{2} = \frac{P_2\,z_2}{2} \quad \text{etc}$$

or

$$\frac{Z_1}{d_1} = \frac{W}{P}, \quad \frac{Z_2}{d_2} = \frac{W}{P_2} \quad \text{etc.}$$

That is to say, the kinematical ratio of a change of length of any member to a resulting component movement of any joint is equal to the ratio between a like component force applied to the joint and the resulting stress in the given member. If it is the vertical component of the movement which is desired, that is to say, the deflection, then the like component of the force at the joint is also vertical and the force itself may be a gravity load. We may then say, for such a case, that the ratio of a deformation of any member to the resulting deflection of any joint is equal to the ratio of any load at the joint to the resulting stress in the given member. That is to say, we may replace an unknown kinematical ratio between (z) and (d) by a known ratio between W and P. This ratio between W and P is

found by the ordinary methods of structural analysis, either algerbric or graphic.

Since it is the deflection of the joint which is sought, eq. (2) may be written,

$$d = \frac{P}{W} z \qquad (3)$$

For convenience let $\frac{P}{W} = u$, when u becomes the ratio of the stress in any member to the load at the given joint which produces it. Since, for any structure, this ratio is constant for all loads, we may make the load at the joint unity, as 1 lb., when it becomes numerically equal to the number of units of stress in the member for a unit load placed at the given joint. Eq. (3) now becomes,

$$d = u\, z \qquad (4)$$

But z is any deformation in any one member for which d, the partial deflection at the joint, is to be found. The elastic deformation of the member, in inches, under the action of a direct force is $z = \frac{p\,l}{E}$, where p is the stress per square inch in the member; l is the length in inches; and E is the modulus of elasticity in pounds per square inch. We may now write,

$$d = \frac{p\,u\,l}{E}, \qquad (5)$$

or, by summing up all the partial deflections we would have, as the total deflection of the joint resulting from the deformation of all the members of the truss,

$$D = \Sigma\, d = \Sigma\, \frac{p\,u\,l}{E} \qquad (6)$$

The calculation of the stresses in structures with redundant members. For this purpose the general formula for deflection, $d = \Sigma \frac{p\,u\,l}{E}$, may be utilized. For this purpose it will

be convenient to replace the unit stress p by the total stress divided by the cross-section of the member, and write,

$$D = \Sigma \frac{S\ u\ l}{a\ E} \tag{15}$$

in which D = deflection or movement of any joint in any given direction due to any loading. s = total stress in any member due to this loading, u = factor of reduction = numerically the stress in the member due to one pound applied at the point whose movement is desired and acting in the given direction, and l,a, and E are length, cross-section and modulus of elasticity of the member.

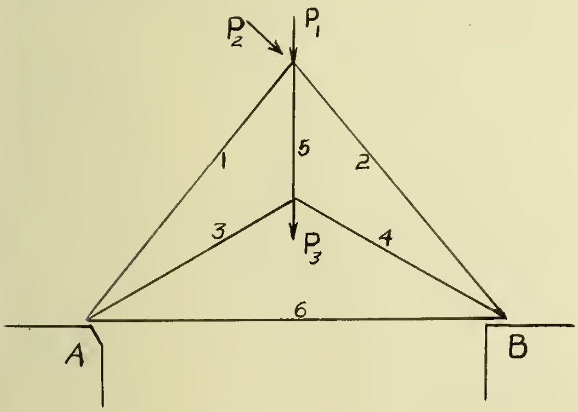


Fig. 1

Let AB represent any s structure loaded in any manner (at joints only) and having one redundant member. In this case any member may be taken for the redundant member; we will take member (6). The truss is hinged at A and supported on rollers at B. Let s_1, s_2, s_3, etc., be the stresses in the several members, as yet unknown. s_6 is the stress in the redundant member. Represent leng lengths, corss-sections, and moduli of elasticity by l_1, l_2, etc a_1, a_2, etc., and E_1, E_2, etc., respectively.

The desired elastic relation can be obtained by considering the movement of B relative to A. Under the loads the point B moves (deflects) towards the right a distance D. If the stresses in all the members were known, or their distortions,

in these members alone would be sufficient data to determine D. The value of D is also evidently equal to the actual elongation of member 6. By the first method we have,

$$D = \Sigma_1^5 \frac{S\,u\,l}{a\,E}, \qquad (a)$$

and by the second,

$$D = -\frac{S_6 l_6}{a_6 E_6} \qquad (b)$$

In (a) note that S is the actual stress in any of the five members, u is the factor of reduction, equal to the stress in the member due to 1 pound applied at B towards the right, the member 6, being removed. The sign indicates a summation of members 1 to 5. In (b) the minus sign is used because tension has been called - and compression .

The deflections from (a) and (b) being identical, we h have,

$$-\frac{S_6 l_6}{a_6 E_6} = \Sigma_1^5 \frac{S\,u\,l}{a\,E} \qquad (16)$$

Now the total stress S in any member may be considered as made up of two parts; (a) a stress which would be caused by the external loads with member 6 removed, and (b) a stress due to forces at A and B equal to the stress in member 6. The first part is readily calculated by the usual methods of statics; call this part S'. The second part, S", can also be calculated when S_6 is known. Noting that u is the stress in any member due to 1 pound acting towards the right at B (reaction at A is also 1 pound), it is evident that S" is equal to S_6u. Hence we may write the general relation,

$$S = S' + S_6 u \tag{17}$$

Substituting this value of S in eq. (16), we have,

$$-\frac{S\,l}{a_6 E_6} = \Sigma_1^5 \frac{S'\,u\,l}{a\,E} + S_6\,\Sigma_1^5 \frac{u^2\,l}{a\,E}$$

whence we derive,

$$S_6 = -\frac{\Sigma_1^5 \frac{S'ul}{aE}}{\frac{l_6}{a_6 E_6} + \Sigma_1^5 \frac{u^2 l}{aE}} \tag{18}$$

The form of expression is evidently the same no matter how many necessary members there may be. If n is the number of such members and r is the redundant member, we may write the more general expression,

$$S_r = -\frac{\Sigma_1^n \frac{S'ul}{aE}}{\frac{l_r}{a_r E_r} + \Sigma_1^n \frac{u^2 l}{aE}} \tag{19}$$

in which it is to be noted that S' is the stress in any of the n members due to the given loads, with the redundant member removed, and u, is the stress due to a force of 1 pound acting at the joints where the redundant member is attached, or it may be thought of as the stress due to 1 pound compression in the redundant member.

The following table gives the quantities necessary for the determination of the stresses in the bents, and the symbols are the same as noted in the preceding discussion, that is:

- l, the length of the member in inches,
- a, the area of the cross-section in sq. inches,
- S', the stress in the member, neglecting the redundant member Y-1. These are found graphically. (See Fig. 2, Plate VII)

PLATE VII

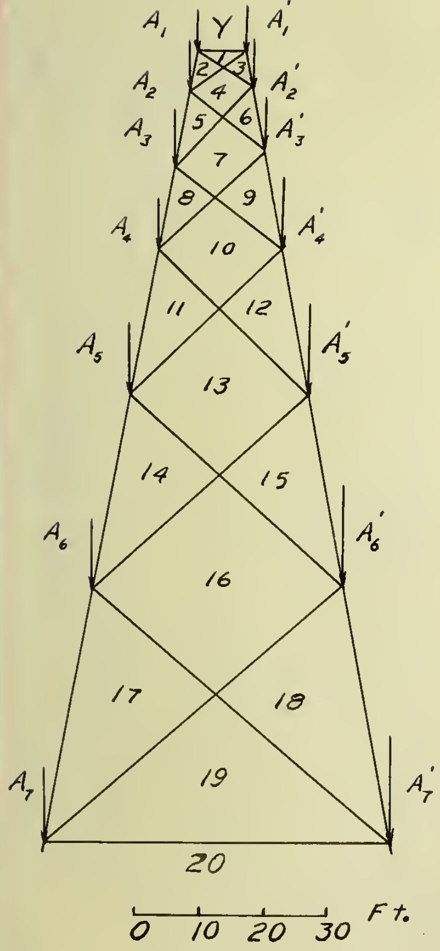
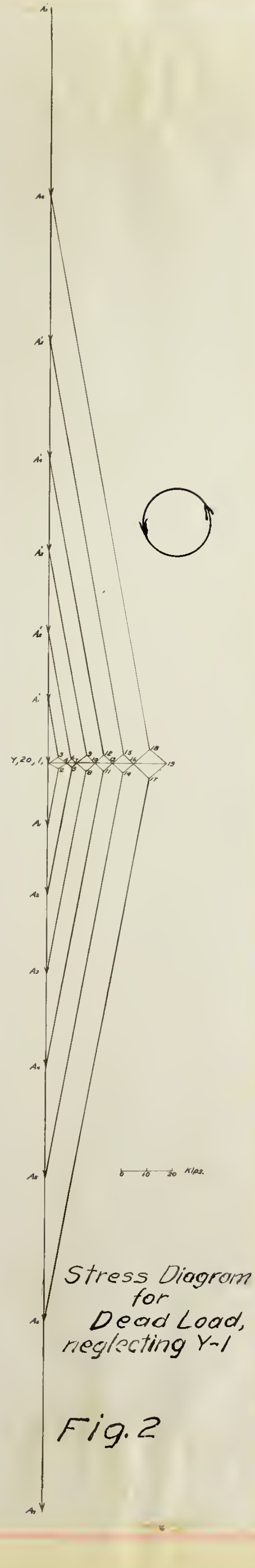
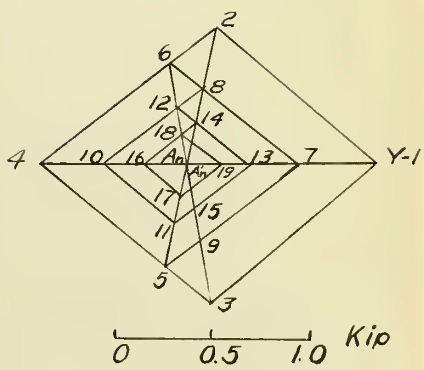


Fig. 1



Stress Diagram for Dead Load, neglecting Y-1

Fig. 2



Stress Diagram for 1 Kip Compression in Member Y-1

Fig. 3

TABLE VIII

Quantities neccessary for determination of stresses in 124 foot transverse bent.

Member	l''	a''	s'	u	$\frac{ul}{a}$	$\frac{s'ul}{a}$	$\frac{u^2l}{a}$	Y-1 x u	S
A_1-2	96"	37.7	-21.5	-0.75	-1.9	+41.0	+1.50	+2.3	-19.7
A_2-5	126	"	-51.0	+0.52	+1.74	-88.5	+0.90	-1.6	-52.6
A_3-8	180	"	-80.0	-0.41	-1.96	+157.0	+0.80	+1.25	-78.8
A_4-11	252	"	-119.0	+0.3	+2.0	-238	+0.60	-0.92	-120.0
A_5-14	348	"	-162	-0.23	-1.8	+292	+0.42	+0.70	-161.3
A_6-17	504	"	-220	+0.15	+1.7	-374	+0.26	-0.46	-220.5
A_1'-3	96	"	-21.5	-0.75	-1.9	+41.0	+1.5	+2.3	-19.7
A_2'-6	126	"	-51.0	+0.52	+1.74	-88.5	+0.9	-1.6	-52.6
A_3'-9	180	"	-80.0	-0.41	-1.96	+157.0	+0.8	+1.25	-78.8
A_4'-12	252	"	-119.0	-0.3	+2.0	-238.0	+0.6	-0.92	-120.0
A_5'-15	348	44.6	-162.0	-0.23	-1.8	+292	+0.42	+0.70	-161.3
A_6'-18	504	"	-220	+0.15	+1.7	-374	+0.26	-0.46	-220.0
1-2	84	11.76	-5.0	+1.1	+7.9	-39.5	+8.7	-3.4	-8.4
2-4	84	"	-5.0	+1.1	+7.9	-39.5	+8.7	-3.4	-8.4
4-5	198	"	-3.0	-0.85	-14.3	+42.9	+12.1	+2.6	-0.4
5-7	198	"	-3.0	-0.85	-14.3	+42.9	+12.1	+2.6	-0.4
7-8	276	"	-5.0	+0.62	+14.5	-72.5	+9.0	-1.9	-6.9
8-10	276	"	-5.0	+0.62	+14.5	-72.5	+9.0	-1.9	-6.9
10-11	384	"	-5.0	-0.47	-15.3	+76.5	+7.2	+1.44	-3.6
11-13	384	"	-5.0	-0.47	-15.3	+76.5	+7.2	+1.44	-3.6
13-14	528	"	-6.0	+0.35	+15.7	-94.2	+5.5	-1.07	-7.07
14-16	528	"	-6.0	+0.35	+15.7	-94.2	+5.5	-1.07	-7.07
16-17	744	"	-10.0	-0.25	-15.8	+158	-3.9	+0.76	-9.3
17-19	744	"	-10.0	-0.25	-15.8	+158	-3.9	+0.76	-9.3
19-20	654	"	+46.0	+0.16	+8.9	+414	+1.4	-0.50	+45.5
Y-1	96	"							
						+345.8	+105.0		

$$\frac{l_{Y-1}}{a_{Y-1}} = 8.1 \qquad Y\text{-}1 = \frac{345.8}{8.1+105} = 3.06$$

U, stress in member due to one kip compression. (See Fig. 3, Plate VII)

Y-1, the redundant member, and

s, the actual stress in the member.

LIVE LOAD, AND COMBINED DEAD AND LIVE LOAD STRESSES IN 124-FOOT TRANSVERSE BENT. The dead load stresses in the transverse bent were determined by graphical resolution. See Plate VII, for stress diagram and Table VIII for stresses. However, as the stresses in the members are directly proportional to the loads that stress them, it will only be necessary to determine the maximum train load that will come upon one cap of the bent, and the loads that will stress the required member. The latter is determined from the stress diagram, Fig 2, Plate VII, and the maximum train load that will come upon one cap of the bent, is determined by considering (1-3), Fig. 2, as two simple beams, and then determining the greatest reaction at 2.

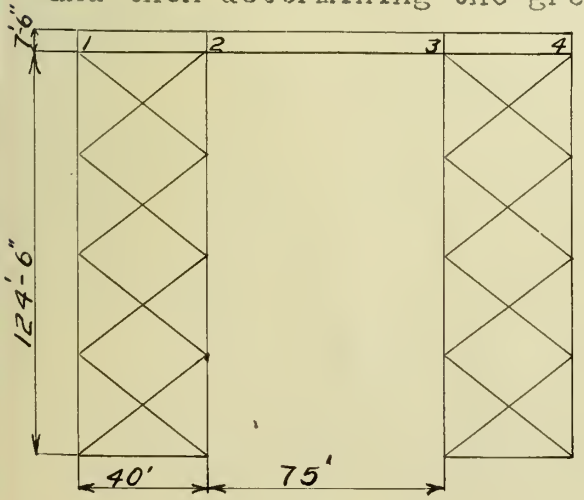


Fig. 2.

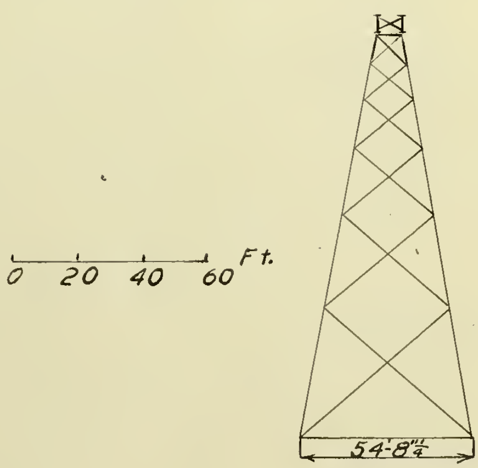


Fig. 3.

Cooper's E.50 Loading is specified, and the loading that will give a maximum for point 2, is that which satisfies the condition,

$$P' = \frac{p'P}{p_1 + p_2}, \qquad (1)$$

where, p, load on 1-3,

p_1, distance 1-2,

p_2, distance 2-3, and

P', load 2-3.

This condition is found to be satisfied when wheel (4) is at 2, then,

$$R_2 = \frac{p_1 M_3 - (p_1 + p_2) M_2}{p_1 p_2} \qquad (2)$$

where, R_2, reaction at 2,

M_3, moment at 3, and

M_2, moment at 2.

The derivation of the formula's (1) and (2) may be found in Merriman and Jacoby's Roofs and Bridges, Part I, pages 126-128, inclusive.

R_2, the maximum train reaction at 2 is found to be 88,812 pounds. So, this amount will be added to each connection, to determine the stresses due to dead and live loads combined. The stresses are given in Table IX.

The second and third columns of the following table give the loads that stress the corresponding member in the first column, the fourth column gives the ratio of the combined dead and live load to the dead load and the last three columns give the stresses.

TABLE IX.

Stresses in 124.5 foot transverse bent.

Member	Dead L.	Dead L. and Live L.	$\frac{D.L. + L.L.}{D.L.}$	D.L.S.	D.L. + L.L. Stress.	L.L. Stress.
A_1-2	24.4	114.4	4.7	-19.7	-56.0	-36.3
A_2-5	26.6	117.0	4.5	-52.6	-236.0	-183.0
A_3-8	31.0	121.0	3.9	-78.8	-318.0	-238.0
A_4-11	37.0	127.0	3.4	-120.0	-410.0	-290.0
A_5-14	45.0	135.0	3.0	-161.3	-484.0	-323.0
A_6-17	57.0	147.0	2.6	-220.3	-520.0	-300.0
1-2	24.4	114.0	4.7	-8.4	-40.0	-32.1
4-5	26.6	117.0	4.5	-0.4	-1.8	-1.4
7-8	31.0	121.0	3.9	-6.9	-28.0	-21.0
10-11	37.0	127.0	3.4	-3.6	-12.2	-8.6
13-14	45.0	135.0	3.0	-7.0	-21.0	-14.0
16-17	57.0	147.0	2.6	-9.3	-24.2	-14.9
19-20	57.0	147.0	2.6	+45.5	+90.0	+44.5
Y-1	24.4	124.4	4.7	-3.06	-14.3	-11.0

STRESSES IN THE LONGITUDINAL BENTS. The longitudinal bracing of each tower must be capable of resisting the greatest tractive force of the engines, and this force will be assumed to act at the point 2, as shown in figure 5. It will also be assumed that this force is divided between the two systems of bracing in one tower. The maximum train load that will come upon 1-3 will be when driver (2) is just to the right of 1. This load is found to be 358000 pounds. Using a friction coefficient of 0.2 the force which the bracing must resist is 0.2 x 358000 = 71,600 lbs; this force is divided between the two systems, so each system must resist 35800 lbs. The stresses are determined and are shown in Figure 6.

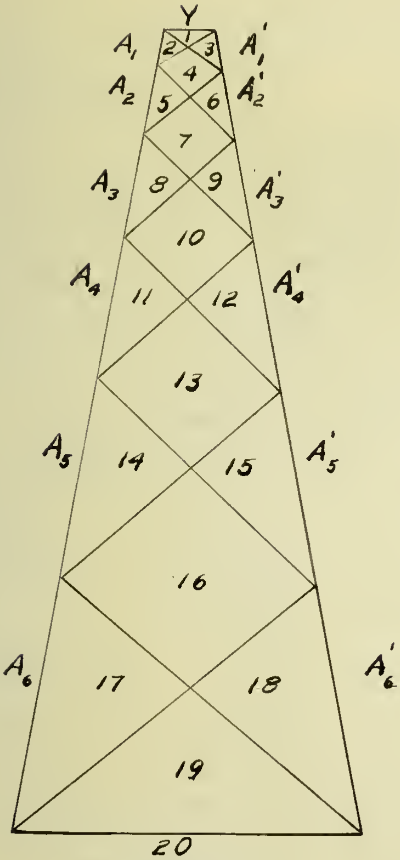


Fig. 4.

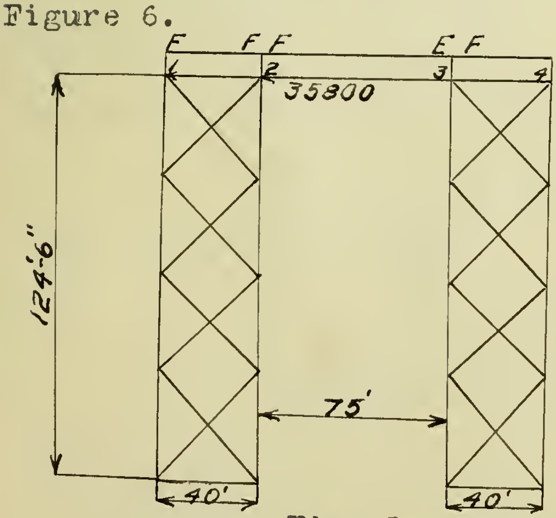


Fig. 5.

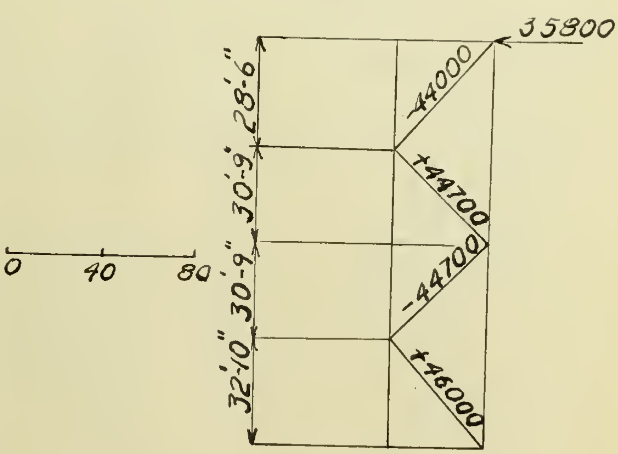


Fig. 6.

The stresses found in the columns were not used in the designing, as trains rarely stop on the bridge. The stresses in the other system of diagonals will be numerically the same but of opposite sign from the corresponding one in the other system.

WIND STRESSES IN TRANSVERSE BENTS. In the transverse bents there is a double system of bracing, as seen in Fig. 1, Plate VIII, thus causing a redundancy of members. For simplification, and as the error will be small, each system is assumed to take one half of the wind forces. With the forces acting as shown in Fig. 1, Plate VIII, the heavier lined system is acting.

The wind forces on the girders, trains, and ties, will be assumed as 30 pounds per square foot, and on the bent, 125 pounds per vertical lineal foot. The determination of the forces are made according to the method of statics, and the results together with the dimensions of the bent are given in Fig. 1, Plate VIII. The towers are forty feet long and the girders on each side are seventy five feet in length. The train is assumed to be ten feet in height, and the ties one foot in depth. The stresses are found by graphical resolution as shown on Plate VIII.

Table X, on the following page, gives the stresses in the members of the bent.

TABLE X.

Wind Stresses in Transverse Bent.

Member.	System acting as shown. Stress.	Both systems acting. Stress.
3-4	-6700	-13400
4-5	+7100	+7100
5-6	-7000	-7000
6-7	+5000	+5000
7-8	-5000	-5000
8-9	+4800	+4800
9-10	-6000	-6000
10-11	+3400	+3400
B-8	+36000	+72000
C-8	+36000	+72000
A-10	+43500	+87000
D-6	+29500	+59000
E-6	+29500	+59000
F-4	+21000	+41000
O-9	-40000	-80000
N-9	-40000	-80000
M-7	-33500	-67000
L-7	-33500	-67000
K-5	-25500	-51000
J-5	-25500	-51000

Wind Load Stresses
in
Tranverse Bent
by
Graphical Resolution

PLATE VIII.

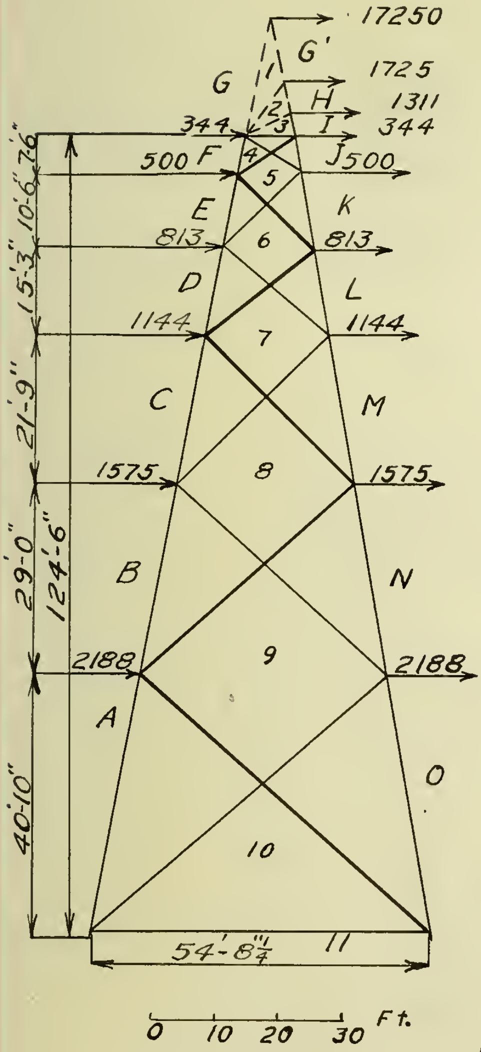


Fig. 1

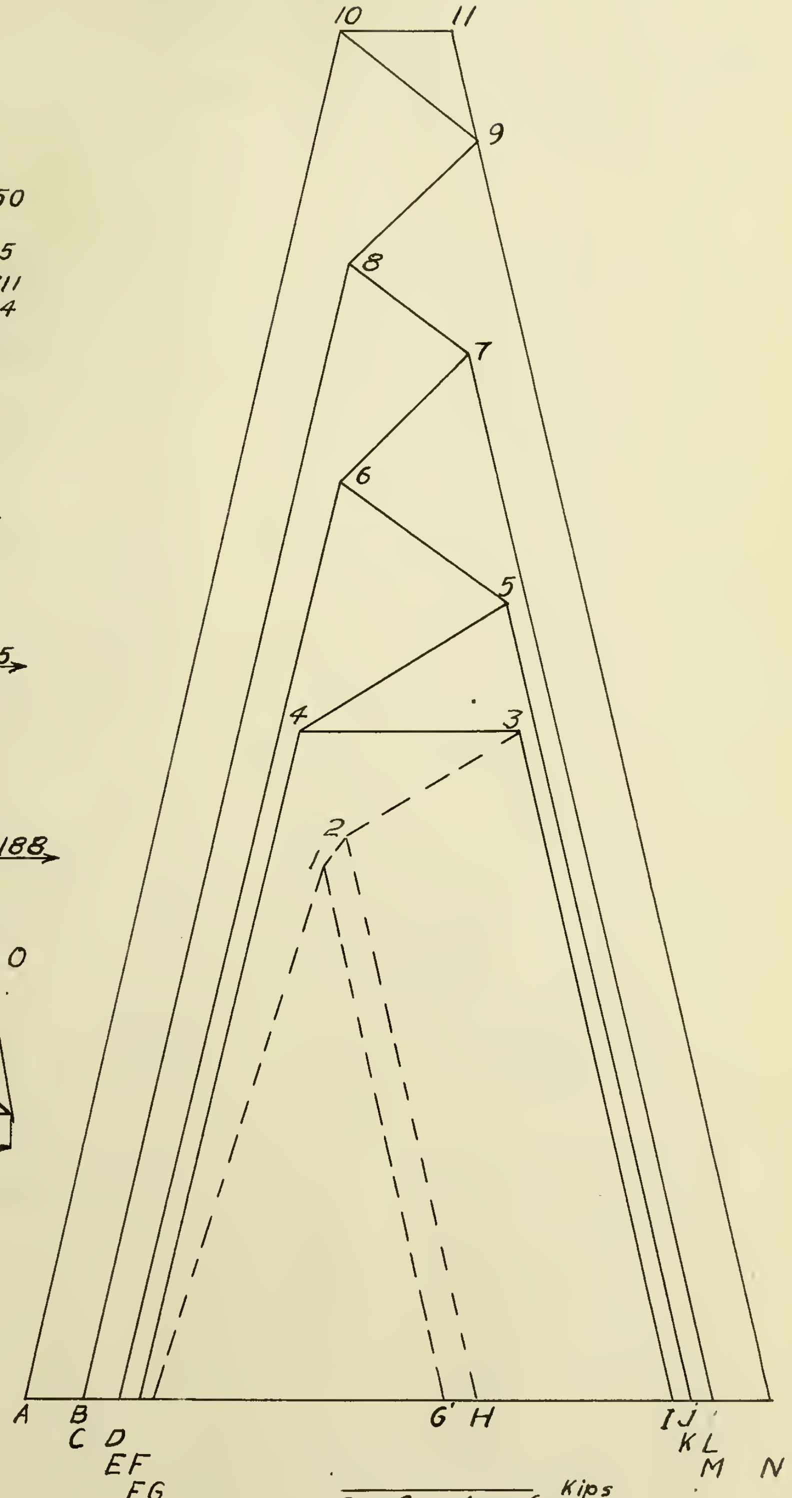


Fig. 2

TOTAL STRESSES IN BENTS. The following tables give all the stresses in the longitudinal and diagonal bents. The impact stress is calculated from the formula $I = L.L. \times \frac{L.L.}{D.L. + L.L.}$ which was discussed in the general description page 3.

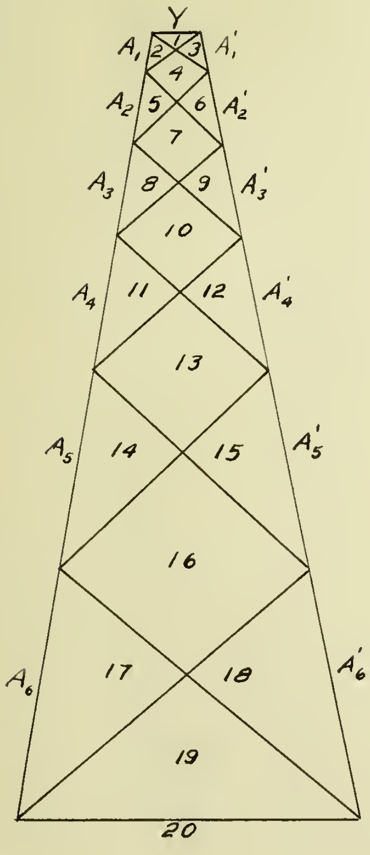


Fig. 7

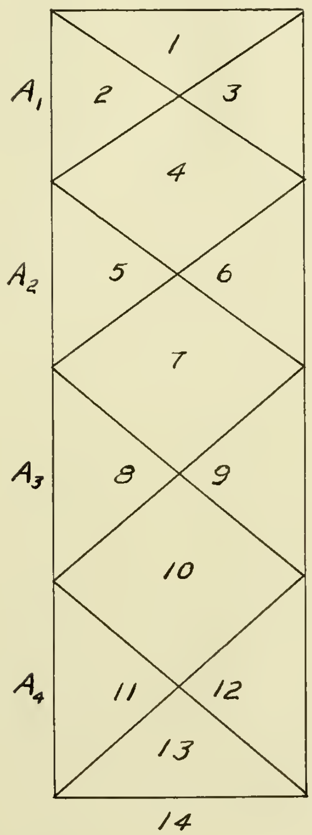


Fig. 8

TABLE XI.

Stresses in diagonal bents.

Member	D.L. Stress.	L.L. Stress.	Imp't.	Wind. S.	Max S.
A_1-2	-19.7	-36.3	-23.0	-42.5	-122.0
A_2-5	-52.6	-183.0	-140.0	-60.0	-436.0
A_3-8	-78.8	-238.0	-178.0	-67.0	-563.0
A_4-11	-120.0	-290.0	-120.0	-67.0	-590.0
A_5-14	61	-323.0	-212.0	-80.0	-776.0
A_6-17	-220	-300.0	-175.0	-80.0	-775.0
1-2	-8.4	-32.1	-26.0	-7.1	-73.0
4-5	-0.4	-1.4	-1.1	-7.1	-10.0
7-8	-7.0	-21.0	-10.5	-5.0	-43.5
10-11	-3.6	-8.6	-6.1	-5.0	-23.6
13-14	-7.1	-14.0	-9.3	-4.8	-35.0
16-17	-9.3	-15.0	-9.3	-6.0	-39.0
19-20	45.5	44.5	22.0	-3.4	115.0
Y-1	-3.06	-11.0	-8.7	-6.7	-29.0

TABLE XII.

Stresses in Longitudinal Bents.

Member.	Stress.
1-2	±44.0
4-5	±44.7
7-8	±44.7
10-11	±46.0
13-14	-71.6

IV.

INVESTIGATION OF GIRDERS AND BENTS.

The webs and flanges of girders were investigated on pp. 7 and 8. See p. 8 and plate VI for sketches of girders and stresses in girder laterals. The laterals are investigated for unit stresses of 15000 pounds per square inch for tension and $\frac{19000}{1+\frac{l^2}{13500r^2}}$ pounds per square inch for compression.

Tables XIII-XV inclusive give the composition, stresses, required areas, actual areas, and efficiencies of the laterals.

All laterals bracking, will be investigated for a unit compressive stress of $\frac{19000}{1+\frac{l^2}{13500r^2}}$, and the columns will be investigated for unit compressive stress of $\frac{15000}{1+\frac{l^2}{13500r^2}}$ pounds per square inch. Where these are al ternate stresses, each stress is considered as increased by an amount equal to 0.8 of the least of the two strains, for determining the sectional areas required. American Bridge Company's Specifications were used.

Table XVI gives the composition, stresses, required areas, actual areas, and efficiencies of members in the diagonal and longitudinal bents. See Fig. 7, p. 23 for a sketch of the tower.

TABLE XIII.

75-foot Girder.

Member	Composition.	Stress	Allowable Stress Tension.	Allowable Stress Compression.	Area Req. sq. in.	Act. A. sq. in.	Eff.
	Top Laterals.						
U_1U_1'	1 ∠ 4"x4"x3/8"	-4100	15000	13100	0.31	2.86	350%
U_2U_2'	"	"		"	"	"	"
U_3U_3'	"	"		"	"	"	"
U_4U_4'	"	"		"	"	"	"
U_5U_5'	"	"		"	"	"	"
U_0U_1'	6x6x3/8	±8625		9800	1.8	4.36	240
$U_1'U_2$	"	±6760		"	1.4	"	310
U_2U_3'	5x5x3/8	±4890		12700	0.75	3.61	480
$U_3'U_4$	4x4x3/8	±3100		10250	0.5	2.86	570
U_4U_5'	"	±1355		"	0.25	"	1140
	Bottom Laterals.						
L_1L_1'	3 1/2x3 1/2x3/8	-1500		9500	0.30	2.48	825
L_2L_2'	"	"		"	"	"	"
L_3L_3'	"	"		"	"	"	"
L_4L_4'	"	"		"	"	"	"
L_5L_5'	"	"		"	"	"	"
$L_0'L_1$	4x4x3/8	±9100		10900	1.5	2.86	190
L_1L_2'	"	±7100		"	"	"	"
$L_2'L_3$	3 1/2x3 1/2x3/8	±5050		9500	1.0	2.48	248
L_3L_4'	"	±3000		"	"	"	"
$L_4'L_5$	"	±1000		"	"	"	"
	End Cross Frames.						
AB	6x6x3/8	±16000		14000	2.3	4.36	190
CD	"	±16000		14000	2.3	4.36	190%

TABLE XIV.

50-foot Girder.

Member	Composition.	Stress	Allowable Stress Tension.	Allowable Stress Compression.	Area Req. sq. in.	Act. A. sq. in.	Eff. %
.	Top Laterals.						
U_0U_1'	1 ∠ 5"x5"x3/8"	±17900	15000	13200	2.66	3.61	135
U_1U_1'	4"x4"x3/8"	-3752	"	13100	0.60	2.86	470
U_2U_2'	"	"	"	"	"	"	"
U_3U_3'	"	"	"	"	"	"	"
U_4U_4'	"	"	"	"	"	"	"
$U_1'U_2$	"	±13200	"	10200	2.6	"	110
U_2U_3'	"	±9100	"	"	1.9	"	150
$U_3'U_4$	"	±5200	"	"	1.0	"	286
	Bottom Laterals.						
L_1L_1'	3 1/2x3 1/2x3/8	-1250	15000	9500	.26	2.48	950
$L_2'L_2$	"	"	"	"	"	"	"
L_3L_3'	"	"	"	"	"	"	"
L_4L_4'	"	"	"	"	"	"	"
$L_0'L_1$	"	±5900	"	9100	1.01	"	246
L_1L_2'	"	±4250	"	"	0.90	"	273
$L_2'L_3$	"	±2520	"	"	0.54	"	490
L_3L_4'	"	±1850	"	"	0.40	"	650
	End Cross Frames.					3	
AB	1 6"x3 1/2"x3/8"	±13000	15000	9500	2.65	3.42	130
DC	"	"	"	"	"	"	"

TABLE XV.

40-foot Girder.

Member	Composition.	Stress.	Allowable Tension.	Stress Compression.	Area Req. sq. in.	Act. A. sq. in.	Eff. %
	Top Laterals.						
U_1U_1'	1∠ 4"x4"x3/8"	-4020	15000	13100	0.68	2.86	420
U_2U_2'	"	"	"	"	"	"	"
U_3U_3'	"	"	"	"	"	"	"
U_0U_1'	5x5x3/8	±13060	"	12700	2.03	3.61	180
$U_1'U_2$	4x4x7/8	±8520	"	10800	1.60	2.86	180
U_2U_2'	"	±4570	"	"	0.80	"	360
	Bottom Laterals.						
L_1L_1'	3 1/2"x3 1/2"x3/8"	-1340	15000	9500	2.3	2.48	108
L_2L_2'	"	"	"	"	"	"	"
L_3L_3'	"	"	"	"	"	"	"
$L_0'L_1$	"	±4350	"	11800	0.72	"	346
L_1L_2'	"	±2640	"	"	0.41	"	600
$L_2'L_3$	"	±870	"	"	0.16	"	1500
	End Cross Frames.						
AB	1 5x3 1/2x3/8	±17000	15000	9600	2.23	3.05	136
CD	"	"	"	"	"	"	"

TABLE XVI.

Diagonal and Longitudinal Bents.

Member	Composition.	Stress	Allow. Stress.	Area Req. Sq."	Act. Area. Sq."	Eff. %
	Diagonal Bent.					
A_1-2	4∠s 4"x4"x9/16" & 2 pls 21" x 1/2"	-122000	14800	8.25	37.72	455
A_2-5	"	-436000	14600	29.9	"	126
A_3-8	"	-563000	14300	39.0	"	97
A_4-11	"	-590000	14000	42.0	"	90
A_5-14	4∠s 4"x4"x5/8" & 2 pls 21" x 5/8"	-776000	12700	60.1	44.70	75
A_6-17	"	-775000	11550	67.0	"	70
1-2	"	+7100 -132000	16500	8.35	11.76	140
4-5	"	+7000 -12500	18000	1.00	"	1176
7-8	"	+45000 -85000	17300	5.6	"	210
10-11	"	+3400 -40000	15500	3.0	"	390
13-14	"	+5000 -63000	13500	5.0	"	233
16-17	"	+6000 -65000	10500	6.7	"	175
19-20	"	+115000	15000	7.7	"	153
Y-1	"	-52000	17000	3.1	"	380
	Longitudinal bent.					
1-2	2 - 10"-20# [s	±44000	9300	8.50	"	140
4-5	"	±44700	"	8.75	"	134
7-8	"	±44600	"	8.70	"	135
10-11	"	±46000	"	9.00	"	131
13-14	"	±71600	"	7.70	"	153

V.

CONCLUSION.

The webs and flanges areas of girders have efficeincies that average 200 to 115 per cent respectively. The member of the lateral systems show large efficiencies, being in some cases 800 per cent. The large excess of area in the laterals is due to the limiting value of $\frac{l}{r}$ equal to 120.

The stresses in the longitudinal bracing are practically the same throughout, each one averaging closely to 45000 pounds. The excess area in these members is about 35 per cent.

In the diagonal bracing, all members but 4-5, average 100 per cent excess area. 4-5 has a stress of 7000 pounds and -12500, and the efficiency is correspondingly large, being 1176 per cent.

The columns of 124' bents are deficient in cross-sectional area, the required area above the splice being 42 square inches and the actual area 37.7 square inches. This gives an efficiency of 90 per cent. Below the splice, the required area is 67 square inches, and the actual area 44.7 square inches. This gives an efficiency of 75 per cent. These columns are built up of, 4 angles, 4" x 4" x 9/16" and 2 plates, 21" x 1/2", above the splice, and of 4 angles 4" x 4" x 5/8", and 2 plates 21" x 5/8" below the splice.

From the above it is seen that all efficiencies are large enough and that the viaduct should withstand all stresses that will be produced by the assumed maximum loads, with the exception of the columns.